3. Structural Depth

3.1 Existing Structural Systems

3.1.1 Geotechnical and Foundation Concerns

The foundation of Quantum III will be constructed on abandoned steel industry facility foundations with fills consisting of silty sand, cinder and slag. With the unpredictability of the subgrade to the deeper bedrock, and the Monongahela River directly adjacent to the building, shallow foundations cannot be used. The fill located deeper in the subgrade has a higher bearing capacity than the aforementioned soils. Therefore, Geo-Mechanics Inc. insisted on 16" diameter auger cast piles with an ultimate load capacity of 300 kips, and design load capacity of 120 kips. Bedrock is located roughly 85 feet below the surface. With the water table resting at 730 ft above sea level—slab on grade is proposed to be at 753'.

Since the building includes no plans for a basement, slab on grade connects with pile caps and grade beams to make up the foundation of QIII. Grade beams line the exterior of the building and connect pile caps where lateral frames are located. Interior gravity columns typically have four piles with a single, separate pile cap, while columns on the exterior wall tie in with grade beams and three- to four-pile configurations. Foundations are 3000 psi concrete with 5000 psi, 16" end bearing 60 ton auger-cast piles. Reinforced concrete grade beams aid in counteracting lateral load uplift underneath the six vertical trusses as well as provide stability around the perimeter of American Eagle Outfitters Quantum III. Foundation stability is a pressing issue given the Monongahela River is but 45' away.

Figure 4 – Ongoing QIII Construction by Monongahela River

3.1.2 Floor Framing

Quantum III is designed for flexibility to allow individual tenants to lay out each floor as they please. It utilizes 30' by 30' bays with a two 'cores' containing elevators, stairs, mechanical openings and bathrooms. Since the extent of the work of the firms stated (Atlantic Engineering Services, The Design Alliance Architects, etc.) was core and shell—the exact placement of partitions is not addressed in the architectural plans as seen in Figure 5 – Typical Architectural Floor Plan.

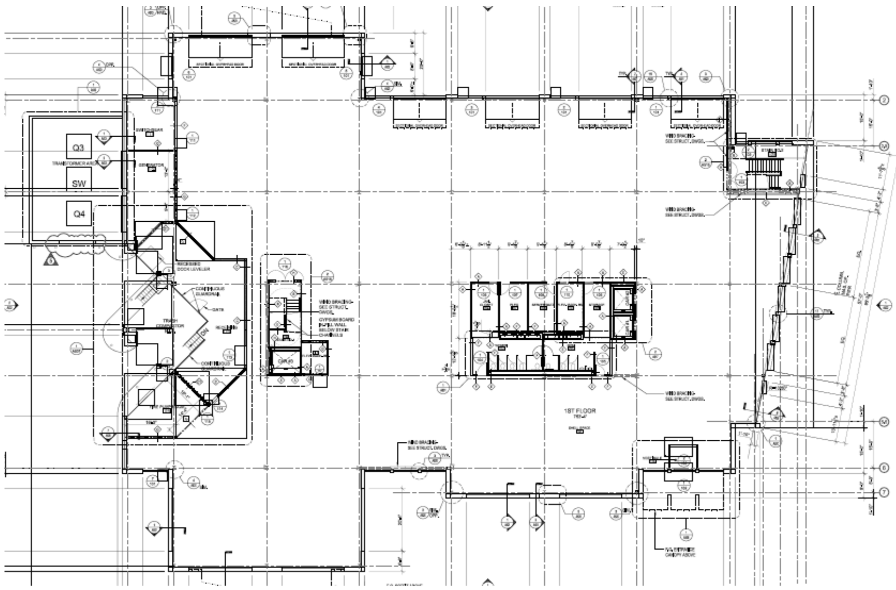


Figure 5 – Typical Architectural Floor Plan

Figure 6 – Typical Floor System Construction

As you can see from the architectural plan, partition placement is not even considered in this stage of the building development. To expand upon the structural system, typical bays for the second through fifth floors are shown below in Figure 7.

All floor framing and steel deck is composite. A lightweight concrete slab on 3" galvanized steel deck was incorporated. Shear studs are 4" long and ¾" diameter in 2.5" lightweight concrete topping. The total slab and deck thickness is 5.5". Typical roof framing consists of 3" metal roof deck, except the mechanical unit area. 2" deck with 3" lightweight concrete provides added support and dampens mechanical vibrations here. Typical girders are W24x55 with 28 studs. Infill beams are W18x35's spaced at 10' center to center with 16 studs. Refer to Figure 7 and Figure 8 for the floor framing layout. American Eagle Outfitters Quantum III has two bays to the north of the building cores as discussed earlier, and one set of bays to the south as seen in Figure 8 – Typical Floor Framing.

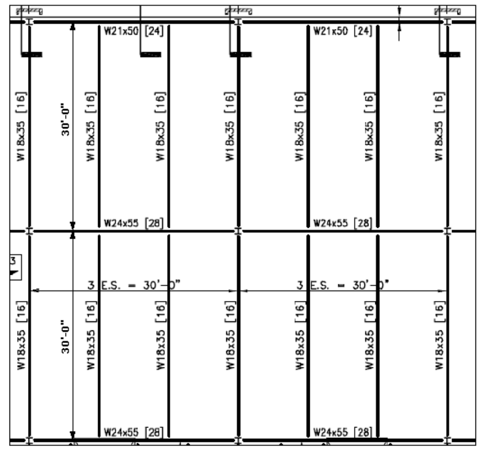


Figure 7 – Typical Bay

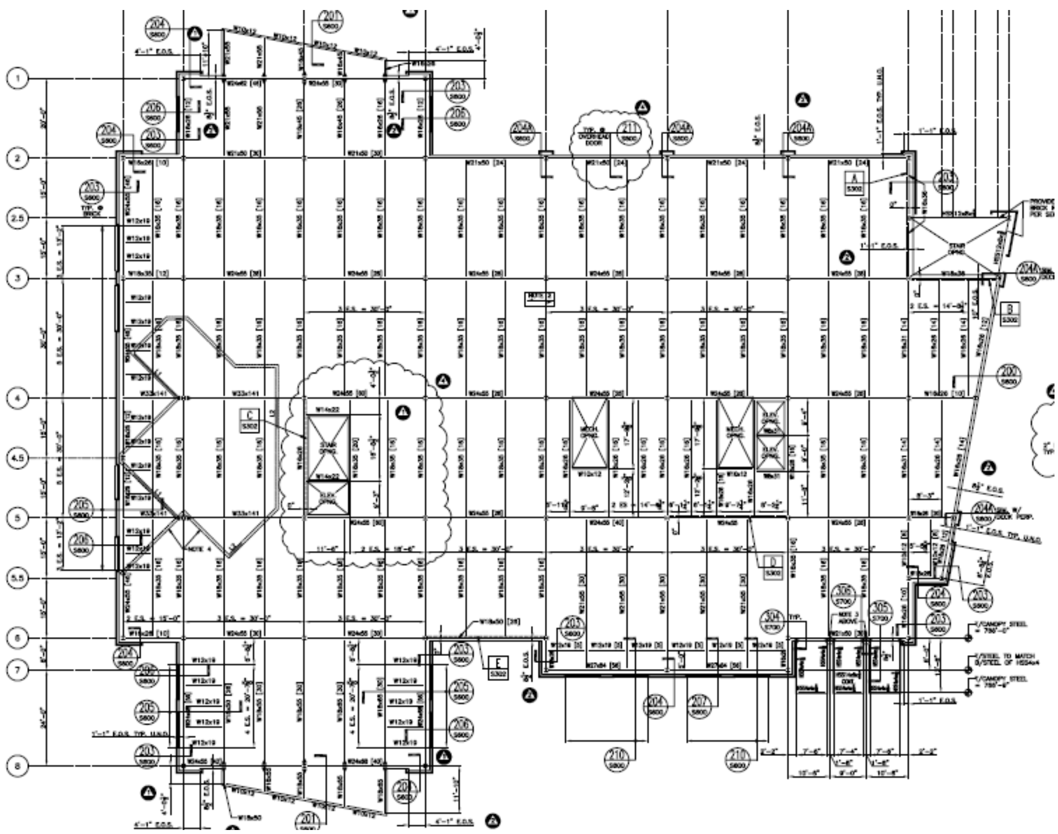


Figure 8 – Typical Floor Framing

3.1.3 Gravity System Columns

Typical columns in AEO: QIII consist of W10's and W12's. Splices are typically located four feet above the top of slab. The fifth floor contains additional columns bearing on transfer beams to support davit pedestals. Columns are placed on a 30' by 30' grid typically.

3.1.4 Lateral Load Resisting Elements

As stated earlier there are five vertical trusses arranged throughout the shell and core of American Eagle Outfitters Quantum III. As shown in Figure 9, their placement was based on resisting interference with the open plan. Also, on the next page are elevations of the vertical trusses in Figure 10 and Figure 12.

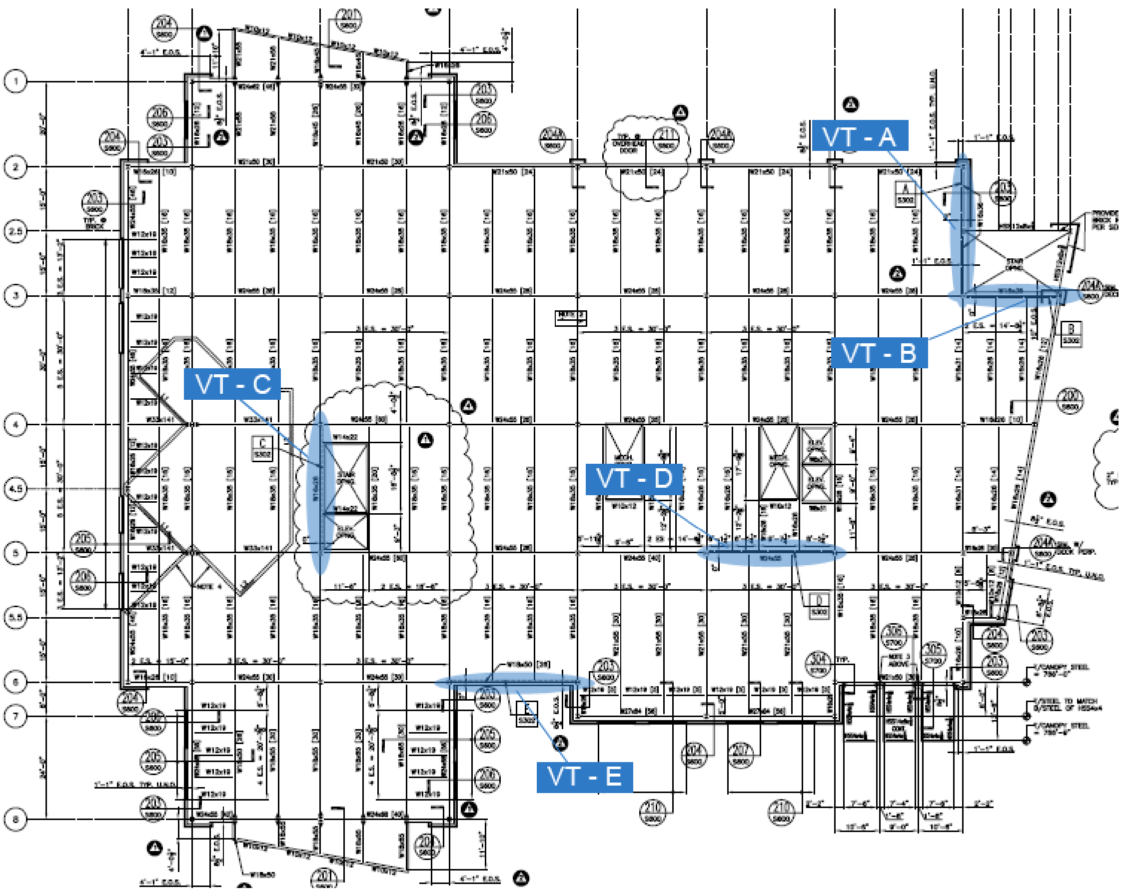


Figure 9 – Vertical Truss Locations

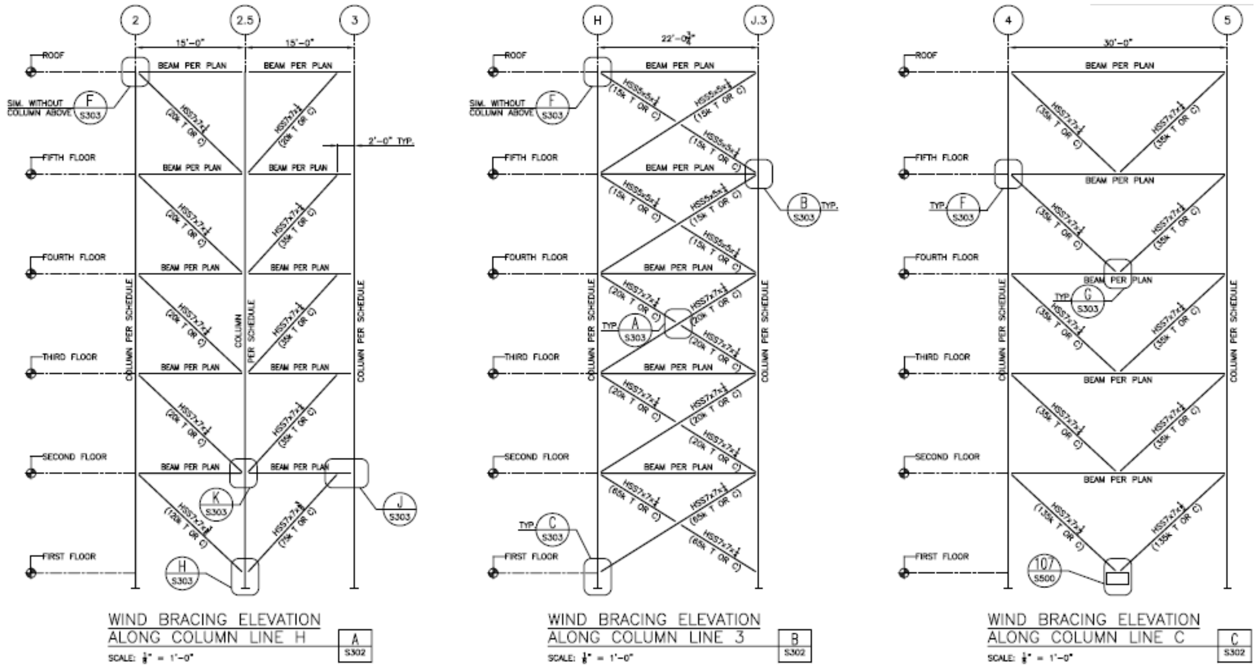


Figure 10 – Vertical Trusses A, B and C (VT-A, B, C)

Vertical truss (VT) A is a single strut truss, VT-B is an X-braced frame, and VT-C is a Chevron truss. VT-A contains an eccentricity to avoid an architectural conflict with stair access doors. All three of the above trusses are located on the interior of the building around stairs, elevators, or mechanical shafts. Braces are HSS7x7's with lateral frame columns ranging from W14x82's to W14x193's. A standard inverted V-truss brace connection is detailed below.

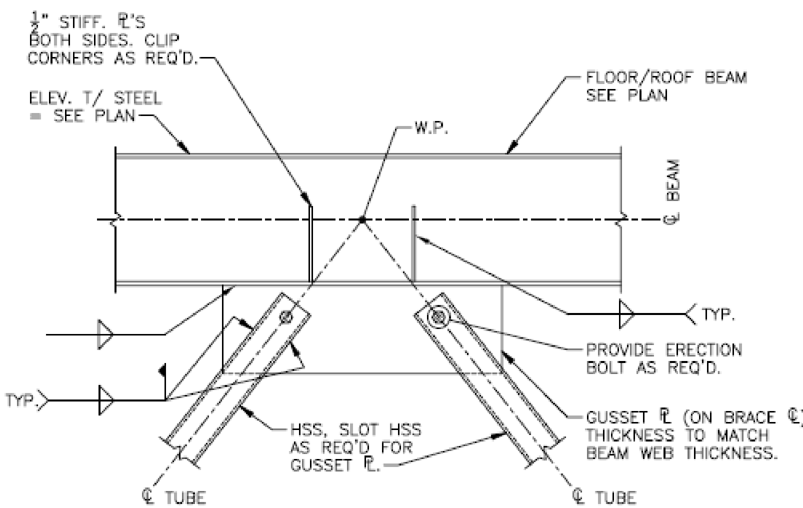


Figure 11 – Brace Connection Detail

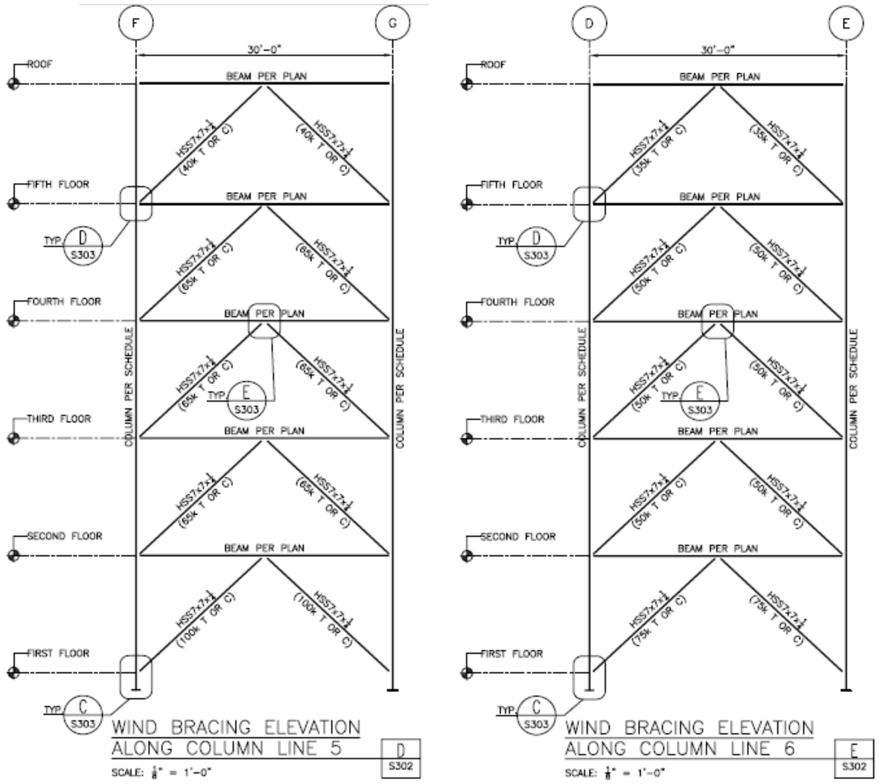


Figure 12 – Vertical Trusses D and E (VT-D, E)

As shown above, VT-D and E are inverted V-trusses. VT-E is the only truss situated on an exterior wall of the building as described earlier.

3.1.5 3-D Model Images

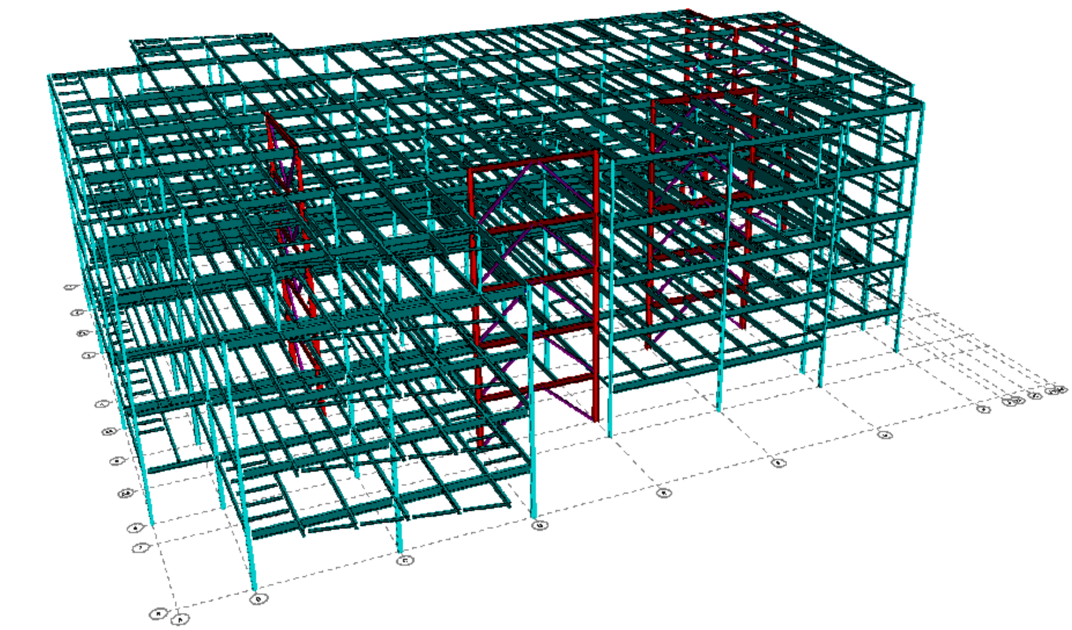


Figure 13 – 3D View from West Building Corner

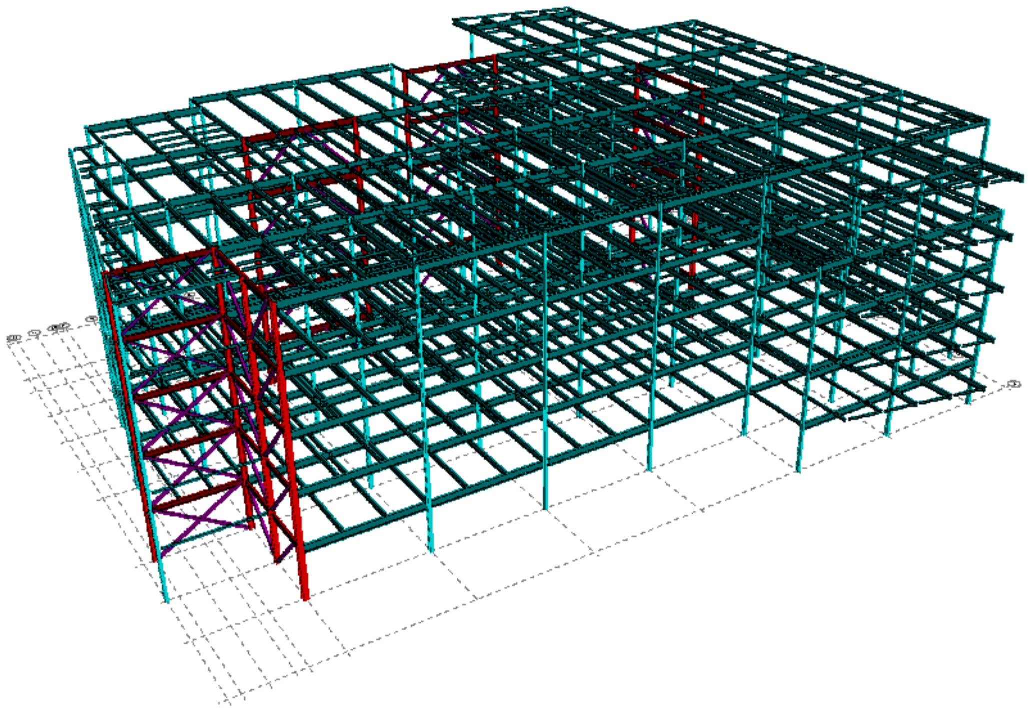


Figure 14 – 3D View from East Building Corner

3.2 Codes and Material Properties

3.2.1 Codes and Referenced Standards

American Eagle Outfitters Quantum III uses the 2003 International Building Code (IBC) as amended by the City of Pittsburgh Building Department. The 2003 IBC references ASCE 7 – 02 and ACI 318-02. All analysis and design was performed by Atlantic Engineering Services using Allowable Stress Design (ASD) as opposed to Load and Resistance Factor Design (LRFD), which is used throughout this technical report. These design methods are prescribed in the AISC Steel Construction Manual, 13th edition, as used for this report.

Codes used for this analysis are IBC 2006 without any Pittsburgh amendments, ASCE 7 – 05 and ACI 318 – 05. Also, California State amendments and Oakland City amendments were analyzed. Upon inspection no amendments directly affected the following analysis.

3.2.2 Material Properties

Concrete

Foundations		3000 psi
Terrace Walls		4000 psi
Interior Slabs		4000 psi
Exterior Slabs		4000 psi
Site Access Canopy Walls		5000 psi
Auger Pile Grout		5000 psi
Reinforcing Steel (Yld)		60 ksi
Headed Concrete Anchors (Yld)	ASTM A108 Grades 1015-1020	60 ksi

Steel

Structural Steel

W Shapes	ASTM A992	50 ksi
M, S, HP Shapes	ASTM A572 Grade 50	50 ksi
Channels	ASTM A572 Grade 50	50 ksi
Steel Tubes (HSS Shapes)	ASTM A500 Grade B	46 ksi
Steel Pipes (Round HSS)	ASTM A500 Grade B	42 ksi
Angles	ASTM A36	36 ksi
Plates	ASTM A36	36 ksi

Galvanized Structural Steel

Structural Shapes and Rods	ASTM A123	
Zinc Coating, Strength of base		
Bolts, Fasteners, and Hardware	ASTM A153	
Zinc coating, Strength of base		
Metal Decking (Yield Strength)		33 ksi
Light Gage Studs, 12-16 Gage	ASTM A653 Grade D	50 ksi
Light Gage Studs, 18-20 Gage	ASTM A653 Grade A	33 ksi

Masonry

Mortar (Prism Strength)	ASTM C270	F'm = 2500 psi
Grout	ASTM C476	F'c = 3000 psi
Masonry (Prism Strength, 28-day)		F'm = 1500 psi

3.3 Existing System Loads and Criteria

3.3.1 Load Cases and Combinations

Below are the load cases considered for Quantum III. Wind and seismic loads were applied in multiple directions to determine the most severe combination. Snow loads were not included in this analysis.

$1.4(D)$
$1.2(D) + 1.6(L) + 0.5(L_r)$
$1.2(D) + 1.6(L_r) + (0.5L \text{ or } 0.8W)$
$1.2(D) + 1.6(W) + 0.5(L) + 0.5(L_r)$
$1.2(D) + 1.0E + 0.5L$
$0.9(D) + (1.6W \text{ or } 1.0E)$

3.3.2 Dead Loads

Unit weights and dead loads are taken from the AISC Steel Manual, 13th Edition. Wall weights are supplied in the structural documents of American Eagle Outfitters: Quantum III. Mechanical unit surface loads described in Figure 16 below are based on an AES design method: distribute two-thirds of the unit weight over one-third the area and the reciprocal distribution of the remaining weight. Of the four distributed loads, the most severe combination is applied to the structure. This assumes most of weight is focused in one section of the mechanical unit and insures QIII is designed for the worst case scenario. The 'opening' refers to the opening for mechanical ducts. Finally, all supporting calculations are available in Appendix A.

Dead Loads			
Component	Typical Floor	Roof	Mechanical Roof
Concrete Slab	38		38
Metal Decking		2	
Flooring/Ceiling	3	4	3
M/E/P	7	10	7
Rigid Insulation		9	
Membrane		2	
Total Dead Load	48	27	48

Figure 15 – Dead Loads

Mechanical Unit Surface Loads								
	2/3 Weight Over 1/3 Area				1/3 Weight Over 2/3 Area			
	With Opening		No Opening		With Opening		No Opening	
Total Weight (lb)	Area (ft^2)	Surface Load	Area (ft^2)	Surface Load	Area (ft^2)	Surface Load	Area (ft^2)	Surface Load
40000	122.5	217.69	225	118.52	272.5	48.93	450	29.63

Figure 16 – Mechanical Unit Surface Loads

3.3.3 Wall Loads

Curtain Walls…………………………………...20 psf (specified in AEO:QIII General Notes)
8" CMU, grout/rein. 24" cc……………...…..51 psf
Partitions……………………………………..20 psf (specified in AEO:QIII General Notes)

3.3.4 Live Loads

The typical bay for the roof has the same dimensions as that for the typical floor, so all reduced live loads are based on the bays and spacing outlined in 3.1.2 Floor Framing.

Location	Load (psf)	Description
Roof	20 **18**	$A_t = 10' \times 30' = 300\ ft^2$ $\therefore R_1 = 1.2 - 0.001A_t = 1.2 - 0.001 * (300\ ft^2) = 0.9$ F = 0, the roof pitch is small enough to be negligible $\therefore R_2 = 1$ $\therefore L_r = R_1 * R_2 * L = 0.9 \times 1.0 * 20 =$ **18 psf**
Offices and corridors above the first floor	80 **54.6** **48.3**	Offices require only 50 psf but since the building is designed to be flexible for tenant fit out, the location of corridors is not currently known, and the conservative corridor load is applied over the entire plan $K_{LL} = 4$: Interior Beams $A_{t,\ beam} = 300\ ft^2$ $A_{t,\ girder} = 15\ ft \times 30\ ft = 450\ ft^2$ $L = L_o \times \left(0.25 + \frac{15}{(K_{LL} \times A_t)^{0.5}}\right) = 80 \times \left(0.25 + \frac{15}{(4 \times 300\ ft^2)^{0.5}}\right) =$ **54.6 psf** $L = L_o \times \left(0.25 + \frac{15}{(K_{LL} \times A_t)^{0.5}}\right) = 80 \times \left(0.25 + \frac{15}{(4 \times 450\ ft^2)^{0.5}}\right) =$ **48.3 psf**
Lobbies and first floor corridors	100	Irreducible per ASCE 7-05 Section 4.8.2
Stairs	100	

3.3.5 Existing Building Wind Criteria

A comparison of wind pressures acting on the main wind force resisting system in Pittsburgh, Pennsylvania is described below. Since the seismic forces in southwestern PA are minimal, wind shears control the design of the lateral force resisting systems. The wind criteria determined for Oakland, California are presented in Appendix B.1.

Assumptions

Building Height (h)	72.33’
Basic Wind Speed (3 second gust)	90
Exposure Category	C
Enclosure Classification	Enclosed
Building Category	II
Importance Factor	1.0
Internal Pressure Coefficient	±0.18
Wind Directionality Factor (Kzt)	0.85
Topographic Factor (Kd)	1.0
Gust Effect Factor (G)	0.84, 0.89

3.3.6 Existing Building Seismic Criteria

Atlantic Engineering Services determined a Seismic Design Category of A for American Eagle Outfitters Quantum III, requiring equivalent lateral forces, F_x, to equal one percent of the total dead load assigned to or located at Level x. They arrived at this conclusion by obtaining different mapped spectral response accelerations of $S_S = 0.131$ g and $S_1 = 0.058$ g**.** This carried throughout the entire seismic calculation, resulting in $S_{DS} = 0.1$ g and $S_{D1} = 0.06$ g—values small enough to qualify for a seismic design category of A. This can be attributed to differing latitude and longitude measurements. In this analysis, Google Earth was used to compute the latitude and longitude of QIII, which resulted in a seismic design category of B. The vertical truss analysis uses category B.

Occupancy Category	II
Seismic Use Group	II
Importance Factor (I)	1.0
Latitude and Longitude	40°25’32.71” N 79° 57’50.93” W
Mapped Spectral Response Accelerations	
$S_s = 0.125$ g	
$S_1 = 0.049$ g	
Site Class	D
Site Class Factors	
$F_a = 1.60$	
$F_v = 2.40$	

S_{MS}	0.20
S_{M1}	0.1176
S_{DS}	0.133
S_{D1}	0.0784
Seismic Design Category	B
Braced Frames are a "Steel System Not Specifically Detailed for Seismic Resistance"	
Response Modification Factor (R)	3.0
Over-strength Factor (W_o)	3.0
Deflection Amplification Factor (C_d)	3.0
Seismic Response Coefficient (C_t)	0.02
Period Coefficient	0.75
Seismic Coefficient (Cs)	0.0284
Building Period (T)	0.921
k	1.211

3.4 Basis for Structural Redesign

Evidence of American Eagle Outfitters current expansion is apparent in Pittsburgh, Pennsylvania. In the past few years, AEO has had two corporate expansions, of which Quantum III is the last installment. Michael Sandretto did a study on Quantum II just last year in AE 481W and 482. The fast turnout of additional corporate office buildings lend to the belief that more Quantum structures are on their way.

As a response to the rapid growth, American Eagle Outfitters could propose expanding with a corporate headquarters on the west coast. To save on design costs, a similar building to Quantum III could be constructed in Oakland, California. The new west coast headquarters must consider the large market the office space must tailor to—so two typical floor layouts will be added in QIII's elevation.

(Note this in no way reflects the actual plans of American Eagle Outfitters and is proposed for the sole purpose of this structural depth.)

3.4.1 Gravity System

The floor plan on the new American Eagle Outfitters: Quantum building will also reflect the need for flexibility. Therefore, the dead and live loads applied on QIII will remain unchanged.

3.4.2 Lateral Force Resisting Elements

Given the seismic design considerations of California, a complete redesign of the lateral systems must be carried out. The original QIII design was in Pittsburgh, Pennsylvania; and was controlled by wind. Due to the large seismic induced forces present in California, lateral systems must be scaled up significantly. Column, brace, and girder sections must all increase as well. Special care will be taken in designing the details for the new Quantum building to ensure safety of the occupants in the event of an earthquake.

Moving the building to a new location presents many new factors when considering a lateral system redesign. The possibility of requiring additional vertical trusses will be met considering the effect of each truss on the existing open floor plan. Also, the higher cooling loads necessary in Oakland can result in the rooftop mechanical unit loads being increased. As a result, seismic acceleration and equivalent loads can grow. As with any engineering task, construction economics will be a considerable factor in the redesign of the lateral systems. The redesign of the lateral force resisting system will take account of all these factors throughout the following pages.

3.4.3 Design Goals and Scope

Due to the inherent complexities of moving a building design to a new site, the goal is to reach an adequate preliminary design for the lateral force resisting system. In this respect, building geometry, redundancy, and the development of plastic hinges throughout the vertical trusses will be taken into account. The lateral force resisting systems will be designed based on strength. Additionally, a preliminary drift evaluation under both wind and seismic loads will be determined to solidify the controlling case.

Overall, the scope of this study is to gain an understanding of design methods used in the architectural engineering field. With experience in East Coast design methods, the move to West Coast provides the daunting task of designing lateral systems to resist earthquake induced loads. The three technical reports completed last fall shrink in comparison to this study on a number of issues. With that said, the following pages outline the precautions taken to design a building to resist and withstand earthquake induced forces, not only to allow the safety of building occupants, but those people inhabiting and travelling through neighboring sites.

3.5 Proposed Gravity System

3.5.1 Gravity Framing

As stated earlier, dead and live loads remain unaltered from the previous Quantum III design. The result is a gravity system not unlike the existing structural sandwich. RAM Structural System was used to obtain the preliminary gravity beams, girders, and columns.

Two typical floors, each at 13'-8" were inserted above the fourth floor. The result was the minor increasing of lower level column sections. Also, the sections that were designated as part of the frame system were altered to be gravity members alone. This provided the minimal allowable design for girders and columns entering into the lateral force resisting system, satisfying the requirement for all frame girders to withstand gravity forces neglecting the truss braces. Shown below is a simple comparison of existing versus new gravity members throughout QIII's structure.

Gravity Member Designs							
Level		Column F3		Girder C3-D3		Infill Beam	
Existing	New	Existing	New	Existing	New	Existing	New
Roof	Roof	W12x40	W12x40	W21x44	W21x45		
5th	7th	W12x53	W12x53	W24x55 [28]	W24x68 [24]	W18x35 [16]	W16x31 [18]
4th	6th	W12x53	W12x53				
3rd	5th	W12x72	W12x72				
2nd	4th	W12x72	W12x72				
	3rd		W12x96				
	2nd		W12x96				

Figure 17 – Gravity Member Comparison

3.5.2 Gravity Frame Detailing

At this point, the level of detail in the gravity system is sufficient to conduct a preliminary lateral force resisting system design. To continue with the depth, a certain number of details were neglected because of their minimal impact on the lateral frame design:

1) Torsion of beams and girders eccentrically supporting shell elements
2) Infill beams around floor openings
3) Reinforced exterior masonry walls at the service entrance on the first floor

3.6 Proposed Lateral Frame Design

3.6.1 New Wind Criteria

Oakland, California has different wind criteria which are outlined below. The actual wind force calculations were completed using an Excel spreadsheet adapted from Technical Report 1. They are available in Appendix B.1.

Assumptions

Building Height (h)	96.64' to Roof T.O.S.
Basic Wind Speed (3 second gust)	85
Exposure Category	C
Enclosure Classification	Enclosed
Building Category	II
Importance Factor	1.0
Internal Pressure Coefficient	±0.18
Wind Directionality Factor (Kzt)	0.85
Topographic Factor (Kd)	1.0
Gust Effect Factor (G)	0.85, 0.88

3.6.2 Wind Design Methodology

Wind pressures were determined using Microsoft Excel (1), and then plotted on a 2-D scale model of the building in AutoCAD. Using the inquiry function, the area of building enclosure was determined and multiplied to find equivalent forces (2). The wind forces were lumped at each floor level, and overturning moment and base shear were calculated in Excel based on each floor's height (3). At this point, lumped wind shears were applied on the diaphragm of an ETABS building model (4). Story drifts were then printed from ETABS, and inserted into another Excel spreadsheet that checked they meet serviceability requirements (5). The methodology is outlined below, and the applicable graphs and output for each step of the process is available in Appendix B.1.

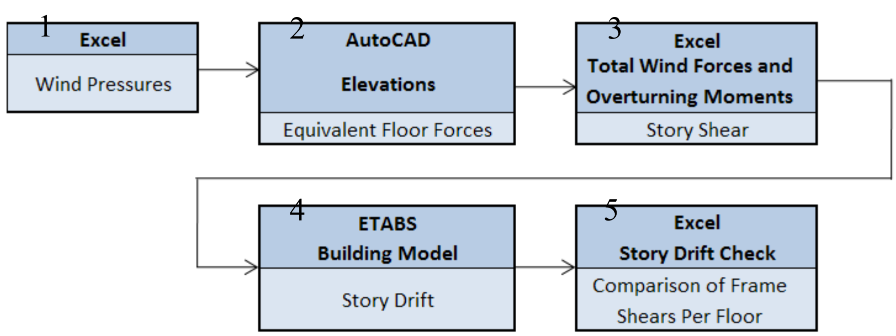


Figure 18 – Wind Analysis Methodology

3.6.3 Wind Story Shears and Overturning Moments

A comparison of North-South and East-West wind was performed to determine which would control story drift. Wind pressures are not assumed to control the strength of lateral force resisting braced frames. Therefore, shears are found to analyze the wind story drift limitation of H/400. Below are the equivalent story shears lumped at each floor level.

Total Wind Forces and Overturning Moments - North-South Wind													
Height Above Grade				Wind Pressure (Windward)	Wind Pressure (Leeward)	Total Wind Pressure	Level	T.O.S. Height	Total Area per Level and Pressure		Force	Total Level Force F (k)	Overturning Moment M (k-ft)
Min	ft	Max	ft						in²	ft²			
0	0	81.3	6.77	9.03	-8.34	17.37	1	0	0	0	0.00	0.00	0.0
81.3	6.77	163	13.5	9.03	-8.34	17.37	2	162.5	214337	1488	25.86	52.66	713.1
163	13.5	180	15	9.03	-8.34	17.37			46165	320.6	5.57		
180	15	240	20	9.60	-8.34	17.94			158280	1099	19.71		
240	20	245	20.4	10.06	-8.34	18.40			11871	82.44	1.52		
245	20.4	300	25	10.06	-8.34	18.40	3	326.5	146409	1017	18.70	56.03	1524.5
300	25	327	27.2	10.45	-8.34	18.79			69907	485.5	9.12		
327	27.2	360	30	10.45	-8.34	18.79			83756	581.6	10.93		
360	30	409	34	11.10	-8.34	19.44			127943	888.5	17.28		
409	34	480	40	11.10	-8.34	19.44	4	490.5	188617	1310	25.47	59.32	2424.70
480	40	491	40.9	11.64	-8.34	19.98			27699	192.4	3.84		
491	40.9	573	47.7	11.64	-8.34	19.98			216316	1502	30.01		
573	47.7	600	50	11.64	-8.34	19.98	5	654.5	72545	503.8	10.06	61.28	3342.25
600	50	655	54.5	12.09	-8.34	20.43			143771	998.4	20.40		
655	54.5	720	60	12.09	-8.34	20.43			172789	1200	24.52		
720	60	737	61.4	12.49	-8.34	20.83			43527	302.3	6.30		
737	61.4	819	68.2	12.49	-8.34	20.83	6	818.5	216316	1502	31.29	60.91	4154.24
819	68.2	840	70	12.49	-8.34	20.83			48473	336.6	7.01		
840	70	901	75	12.85	-8.34	21.19			153597	1067	22.60		
901	75	960	80	12.85	-8.34	21.19	7	982.5	156961	1090	23.09	66.44	5439.84
960	80	983	81.9	13.17	-8.34	21.51			59355	412.2	8.87		
983	81.9	1065	88.7	13.17	-8.34	21.51			230825	1603	34.48		
1065	88.7	1080	90	13.17	-8.34	21.51	Roof	1147	26380	183.2	3.94	34.90	3334.33
1080	90	1147	95.5	13.47	-8.34	21.81	Roof		204445	1420	30.96		
1147	95.5	1303	109	13.47	-8.34	21.81	Roof - Stair	1147	4392	30.5	0.67	87.39	4771.06
1147	95.5	1291	108	30.31	-20.20	50.51	Windscreen		120960	840	42.43		
1147	95.5	1195	99.6	29.82	-19.88	49.70	Parapet		128352	891.3	44.30		
											Totals	478.92	25704.0

Figure 19 – North-South Wind Shears and Overturning Moments

Total Wind Forces and Overturning Moments - East-West Wind											
Height Above Grade		Wind Pressure (Windward)	Wind Pressure (Leeward)	Total Wind Pressure	Level	T.O.S. Height	Total Area per Level and Pressure		Force	Total Level Force F (k)	Overturning Moment M (k-ft)
ft	ft						in^2	ft^2			
0	6.77	9.44	-8.91	18.35	1	0	0	0	0.00	0.00	0.0
6.77	15	9.44	-8.91	18.35	2	162.5	224188	1556.9	28.57	48.90	662.1
15	20	10.52	-8.91	19.43			139920	971.67	18.88		
20	20.38	10.93	-8.91	19.84			10494	72.875	1.45		
20.38	25	10.93	-8.91	19.84	3	326.5	129426	898.79	17.83	53.88	1466.1
25	30	11.61	-8.91	20.52			139920	971.67	19.94		
30	34.04	11.61	-8.91	20.52			113102	785.43	16.12		
34.04	40	11.61	-8.91	20.52	4	490.5	166738	1157.9	23.76	55.33	2261.79
40	47.71	12.17	-8.91	21.08			215710	1498	31.57		
47.71	50	12.17	-8.91	21.08	5	654.5	64130	445.35	9.39	57.14	3116.78
50	60	12.64	-8.91	21.55			279840	1943.3	41.89		
60	61.38	13.06	-8.91	21.97			38478	267.21	5.87		
61.38	70	13.06	-8.91	21.97	6	818.5	241362	1676.1	36.83	58.72	4005.06
70	75.04	13.43	-8.91	22.34			141086	979.76	21.89		
75.04	80	13.43	-8.91	22.34	7	982.5	138754	963.57	21.53	61.93	5070.78
80	89.17	13.77	-8.91	22.68			256520	1781.4	40.40		
89.17	90	13.77	-8.91	22.68	Roof	1146.5	23320	161.94	3.67	32.53	3107.67
90	96.46	14.08	-8.91	22.99			180730	1255.1	28.85		
96.46	100	14.08	-8.91	22.99	Roof - Stair	1146.5	6120	42.5	0.98	58.39	3187.72
100	109.5	14.63	-8.91	23.54			12750	88.542	2.08		
96.46	108.5	30.31	-20.20	50.51	Windscreen		43350	301.04	15.21		
96.46	108.5	30.31	-20.20	50.51			2550	17.708	0.89		
96.46	100.5	29.82	-19.88	49.70	Parapet		113664	789.33	39.23		
									Totals	426.83	22878.0

Figure 20 – East West Wind Shears and Overturning Moments

As you can see in Figure 19, North-South wind forces are greater, and will control the wind drift check of American Eagle Outfitters: Quantum III. A conservative estimate of the building weight resulted in a factor of over 60 against overturning. This is due to the large volume of the building in comparison to the surface area wind can act on. The overturning calculation is available in Appendix B.2.

3.6.4 Wind Induced Story Drift

The story drift of Quantum III as a result of wind induced forces was minimal at most. Since wind was not assumed to control story drift or strength design of the vertical trusses, they were designed using seismic loads. After a satisfactory preliminary design was achieved in ETABS, wind forces were applied on the model and drift was calculated. The minimum allowed story drift was equivalent to 0.40625 inches at the first floor. With large seismic force resisting vertical trusses, wind induced drift was limited to less than 1/1000th of an inch for a single story. This reinforces the assumption that seismic forces not only control the design of the lateral system but dominate it. The study of wind forces on AEO: QIII did not progress beyond this stage to allow ample time to analyze the complexities of earthquake induced forces.

3.6.5 New Seismic Criteria

As shown below, the seismic coefficients for California vary greatly from that of Pennsylvania. In order to meet code requirements for seismic design category E, the AISC Seismic Design Manual was used. Since American Eagle Outfitters: Quantum III contains both eccentrically braced frames and concentric braced frames, the conservative Response Modification Factor, Over-strength Factor, and Deflection Amplification factors were used. These values were that for special steel concentric braced frames. Supporting calculations are in Appendix B.3.

Occupancy Category	II
Seismic Use Group	II
Importance Factor (I)	1.0
Location	12th St., Oakland, California
Mapped Spectral Response Accelerations	
$S_s = 1.522$ g	
$S_1 = 0.6$ g	
Site Class	D
Site Class Factors	
$F_a = 1.\ 0$	
$F_v = 1.5$	
S_{MS}	1.522
S_{M1}	0.9
S_{DS}	1.015
S_{D1}	0.6
Seismic Design Category	E
Braced Frames are “Special Steel Concentric Braced Frames”	
Response Modification Factor (R)	6
Over-strength Factor (W_o)	2.0
Deflection Amplification Factor (C_d)	5.0

Seismic Response Coefficient (C_t)	0.02
Period Coefficient	0.75
Seismic Coefficient (C_s)	0.1054
Building Period (T)	0.949

3.6.6 Additional Lateral Frames

From the start of the lateral system redesign it was understood that the five frames present throughout American Eagle Outfitters: Quantum III will not be sufficient for seismic forces. To provide for redundancy and achieve an adequate preliminary design, a number of locations for additional braced frames were investigated. Existing vertical trusses are designated with a VT and potential new trusses are designated with an NT. See Figure 21 below.

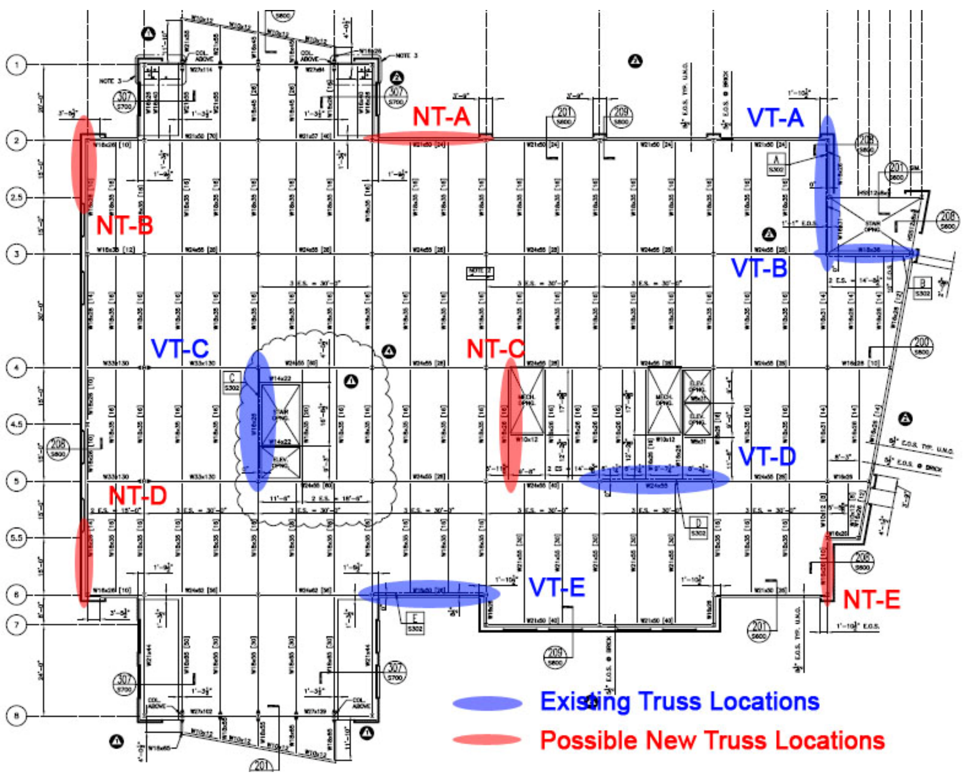


Figure 21 – Existing and Potential New Vertical Truss Locations

Most direct shear will be taken by the most rigid frames, so VT and NT-C would dominate the design in the y direction. VT-D, E, and NT-A are all 30' span trusses and will provide excellent shear resistance and redundancy in the x direction. NT-B, D, and E all span 15', and are therefore less efficient to resist story shears. However, their placement on the building shell maximizes their ability to resist torsional shears. Because the lateral force resisting systems are placed so asymmetrically, there exists the possibility of torsional irregularities. Not only could this increase the apparent seismic forces on the building through the redundancy factor and torsional shears, but can cause equivalent lateral force analysis to be not permitted.

3.6.6.1 Vertical Truss Elevations

As shown to the right, the proposed trusses B, D and E are slimmer, reducing their efficiency in resisting story shears. The X-bracing scheme also is inefficient in the number of connections it requires. On a per story basis, an X-braced frame requires five connections to be detailed whereas an inverted V-truss such as NT-A and C require only three. In a seismic controlled region such as Oakland, California; the detailing would vastly increase the building cost.

To combat the amount of detailing required NT-B, D, and E should be changed to inverted V-trusses beyond this preliminary design. In addition, the elevations below demonstrate the need for foundation detailing at the base of NT-B and D. They appear to be "floating". Be assured this is not the case; the slab on grade is directly below the end of the truss outlined in blue. Therefore, the walls shown below are a combination of structural and retaining walls. Special reinforcing details are required to insure shear and axial forces are transferred to foundations and piles. (Note: Image below is of original QIII elevation and is used to demonstrate the foundation requirements below trusses NT-B and D.)

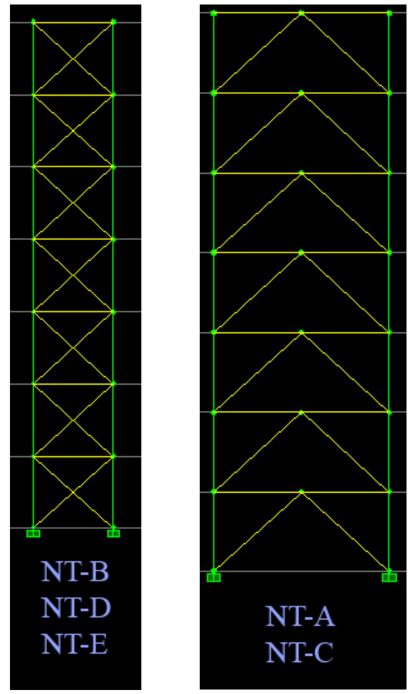


Figure 22 – Proposed Truss Elevations

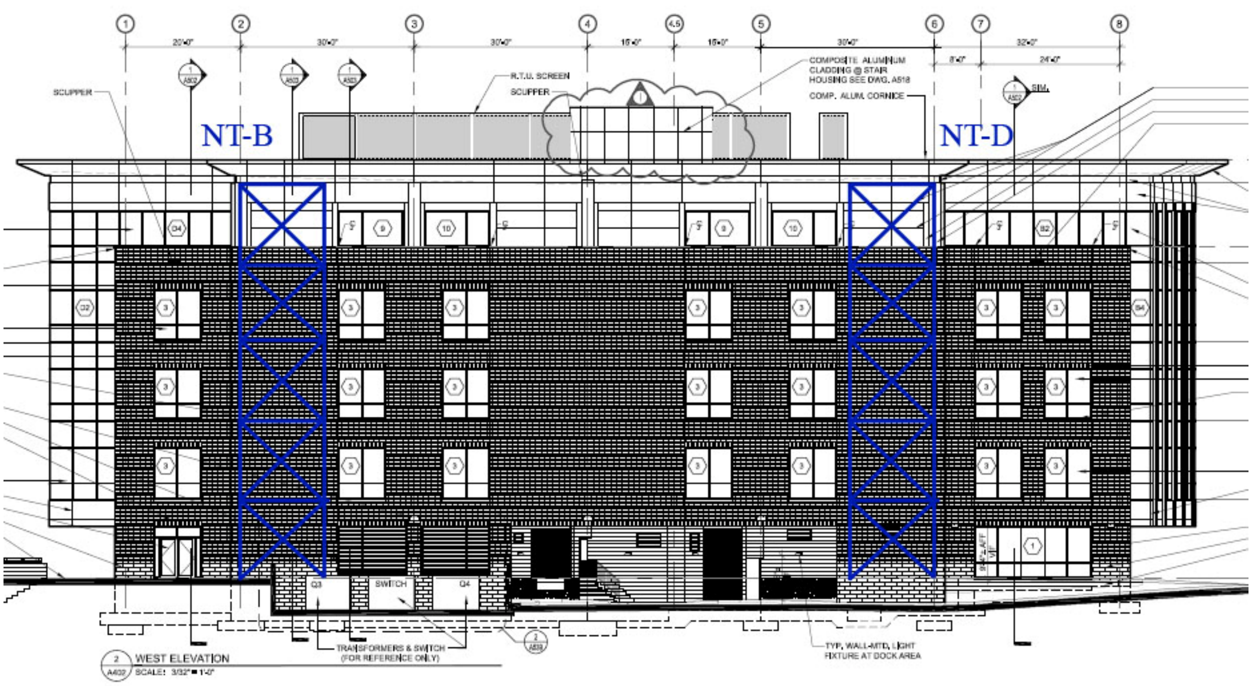


Figure 23 – West Elevation and NT-B and D

3.6.7 Seismic Design

A number of differing methodologies were employed in determining frame location and sizes for QIII. To get to the current preliminary design, the author went through over five possible designs of the lateral system, and with each iteration, discovering more efficient design methods. All methods employed RAM Structural System for story weights and SCBF beam gravity designs. Excel was used to determine equivalent seismic story forces. These forces were then compared to ETABS calculated results. Each method diverged in its approach to design the lateral system after this point. These anomalies in approach are outlined in Sections 3.6.7.2 and 3.6.7.3.

3.6.7.1 Seismic Story Shears

Utilizing story weights obtained from an updated RAM Structural System Model, equivalent seismic story forces and shears were found. By applying the respective building period and seismic coefficient (C_s), the forces, story shears, and overturning moments shown below were obtained. Also, the Excel and hand calculations were compared to ETABS model results shown in Figure 25.

Seismic Base Shear										
Level	h_x (in)	h_x (ft)	h_x^k	W	W * h_x^k	C_{vx}	F	V	M	∑M
Roof	1146.50	95.54	265.917	1420	377655.3	0.146	311.34	311.34	29745.96	29745.96
7	982.50	81.88	220.117	3140	691057.6	0.267	569.71	881.05	46645.01	93290.02
6	818.50	68.21	176.009	3136	551963	0.213	455.04	1336.09	31037.52	124327.5
5	654.50	54.54	133.852	3141	420361.3	0.162	346.55	1682.64	18901.26	143228.8
4	490.50	40.88	94.022	3143	295511.5	0.114	243.62	1926.26	9957.992	153186.8
3	326.50	27.21	57.121	3148	179809.8	0.069	148.24	2074.49	4033.249	157220
2	162.50	13.54	24.307	3155	76683.93	0.030	63.22	2137.71	856.0834	158076.1
1	0.00	0.00	0.000	0	0	0.000	0.00	2137.71	0	158076.1
			Totals	20281.9	2593043	1	2137.71		141177.1	

		Cs	W (kips)		Total Force	
V = C_s * W =		0.1054	20281.9	=	2137.71226	k

T	k
0.50	1
0.95	1.2245
2.50	2

	Lower Bound	Exact	Upper Bound	Use
Cs =	0.05	0.169	0.1054	0.1054

Figure 24 – Seismic Base Shears

Seismic Base Shear Comparison			
Level	Hand Calculated k	ETABS k	Percent Difference
Roof	311.34	327.1	4.82
7	881.05	917.92	4.02
6	1336.09	1391.37	3.97
5	1682.64	1755.67	4.16
4	1926.26	2013.2	4.32
3	2074.49	2170.67	4.43
2	2137.71	2238.14	4.49

Figure 25 – Seismic Base Shear Comparison

3.6.7.2 Design A

Elevation and Framing

The layout used for the first design included all existing trusses as well as NT-B, C, and D. To place NT-C, columns moved less than 6' to be flush with the mechanical space opening shown in Figure 21. Beams that framed into this column were slightly elongated or shortened and had minimal effect on the beam design or structural sandwich.

Methodology

The first design involved trial and error through sizing and resizing frame members in ETABS. As expected, there are many faults with this approach. First, the systematic increasing of member sections to resist lateral loads proved to be fundamentally flawed. After adding NT-B and D, all y axis frame sections were simultaneously increased. In effect, by increasing the column sections of VT and NT-C, their stiffness increased as well. Therefore more seismic shear was distributed to this frame. This resulted in ever-increasing section sizes, never producing an adequate framing layout.

At this point in study, it was found that taking a counter-intuitive approach to lateral design was necessary. By downsizing the most rigid braced frame, more story shear is filtered to, in this case, NT-B and D. When all members finally passed the preliminary ETABS design, most columns for exterior wall trusses were a staggering W14x730. Conversely, interior truss column sections were W14x370 or smaller. When lateral frame dead and live loads were applied, these interior column sections were too small for combined loading. At this point, this design method was proved inadequate and other means were pursued.

3.6.7.3 Design B

This design on American Eagle Outfitters: Quantum III was the most in depth analysis performed for the structural depth. It utilized Excel spreadsheets, ETAB's, and RAM Structural System to get preliminary frame member sizes based on criteria outlined in *Methodology*.

Elevation and Framing

Due to the high relative stiffness of frames VT and NT-C and the apparent gravity loads, these trusses proved inadequate for preliminary design. If sections increased, more shear force would cause them to fail; decreased sections meant failure under gravity loads and minor combination loading. Therefore, both of these were removed. The remaining frames in Design B are shown below.

Figure 26 – Design B Frame Locations

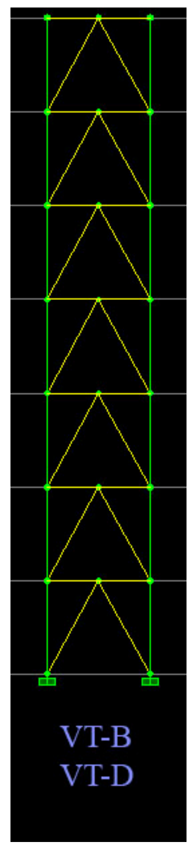


Figure 27 – Design B VT-B and D Elevation

V-trusses are researched as an alternative for X-braced frames NT-B and D due to the increased number of connections required. At 15' long, the member sizes and number of connections required for X-braces create a massive frame that is not efficient or economic. Inverted V-trusses interrupt vertical load paths of the braces and therefore require more shear strength in beams. The author believed this to be an adequate sacrifice to avoid more connection details. The elevation for NT-B and D is at right.

Methodology

Design B utilized the full design process shown below to achieve a preliminary lateral framing design for American Eagle Outfitters: Quantum III. The flowchart has step by step descriptions and Appendix B.3 has each spreadsheet utilized in Design B. Had more time been available, further analysis would be performed. Further considerations past what is covered in this methodology is outlined in 3.6.7.4.

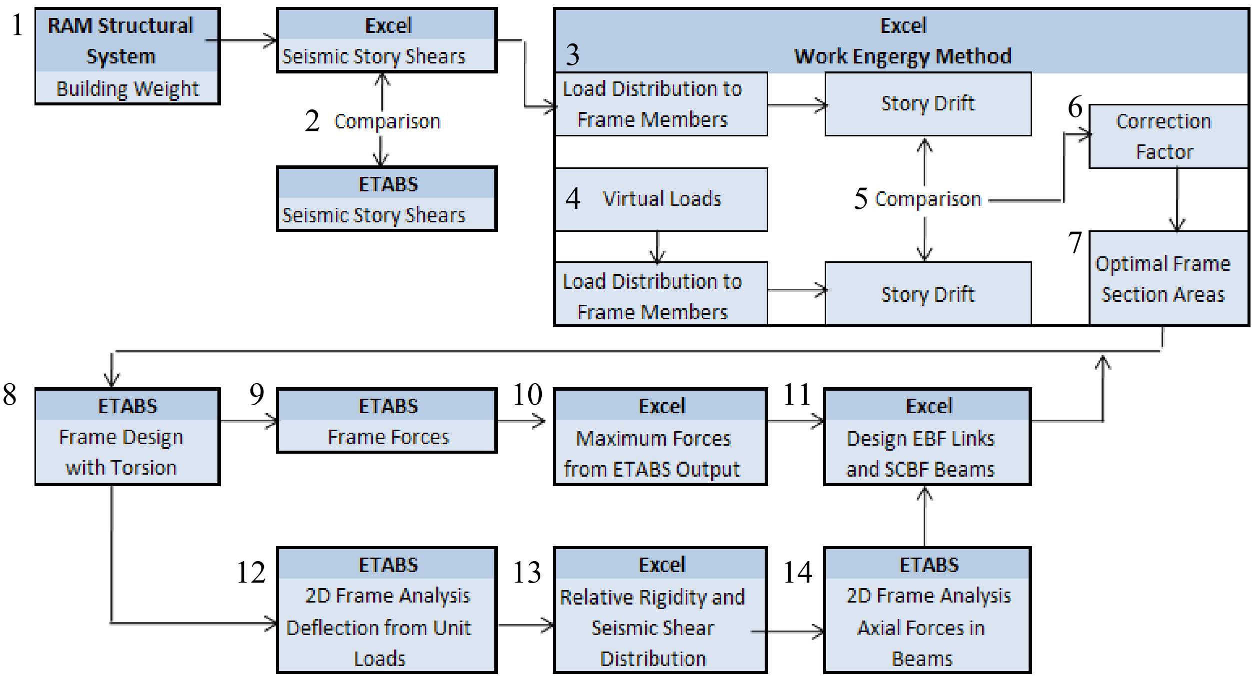


Figure 28 – Design B Methodology

As outlined previously, RAM Structural System was used to find story weights and add them into Excel (1). Story shears, calculated in Excel, were compared to those found in ETABS (2). The seismic shear forces, determined from Excel, were then divided by the number of trusses acting in each orthogonal direction. For frames running in the x-direction in Figure 26, total seismic story shear was divided by three. This assumes each frame is equally rigid and neglects torsion. For frames running in the y-direction, the seismic story shear was divided by two. NT-B and D are significantly less rigid and therefore provide less resistance to seismic shears as VT-A (3).

Using work-energy method, preliminary column sizes were found based on allowable drift. An Excel spreadsheet was developed to analyze virtual loads acting on each vertical truss, and calculate their expected story drift (4). The members optimal, cross sectional areas were then determined based on their allowable seismic drift and equivalent lateral forces through a correction factor (5-7). An example spreadsheet for this procedure is available in Appendix B.3.

The required frame sections were then put into an ETAB's model, and torsional effects were taken into consideration. Utilizing strength design, all members were sized against the 50 load cases ETABS considered (8). Frame forces were then input to Excel, which would locate the maximum shear and moment on beams (10). Frame designs were inserted to another,

separate ETABS model to find frame beam axial forces (12-14). Finally, utilizing more Excel spreadsheets, eccentric brace frame (EBF) links and special concentric braced frame (SCBF) beams were designed (12). The last steps (8-14) were an iterative process to optimize the design.

Results

The truss elevations to the left and on the next page display the wide flange sections used for Quantum III's lateral force resisting system. It was found that the effectiveness of a SCBF was attributed to: 1) its column sizes, 2) brace strength, and 3) beam size. It was in this order that frame sections were designed. Due to local buckling issues, only certain wide flange sizes could be used in seismic regions. The frames contain all allowable wide flange shapes as outlined in the AISC Seismic Design Manual. Utilizing ETAB's, braces were optimized through numerous iterations of the framing layout and member sizes.

The presence of W14x426's reinforce the author's belief on NT-B and D: their half-bay length greatly reduces the efficiency of the frame. With a smaller moment arm to each column, the bending force each truss can withstand is severely decreased. Larger member sections are needed to achieve the same strength as a full-bay length.

Large beam sizes are the direct result of brace sizing. With inverted V-trusses, beams must be designed to withstand 100 percent of the tension brace yield strength and 30 percent of compression brace nominal strength. The result is a large magnitude vertical force on the beam. In this design, shear forces could exceed 1000 kips.

As with shear forces, a beam's strength is determined by the area of the web alone. It is required that shear reinforcing is placed within the web to increase the cross sectional area resisting the shear forces. This will lead to an economic frame girder design. Another obvious fix for this problem is to allow members to transfer that vertical force on the beam, i.e. make the frame have multi-story X-braced frames. Continuous load paths transfer seismic force throughout the frame, allowing all members to supply their full cross sectional area for strength. By continuing design in this fashion, the uneconomic design of the beams shown in Figure 30 and Figure 31 can be eliminated. Figure 29 displays the stress ratio key for all frame elevations.

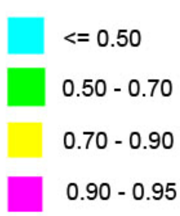


Figure 29 – Stress Ratio Key

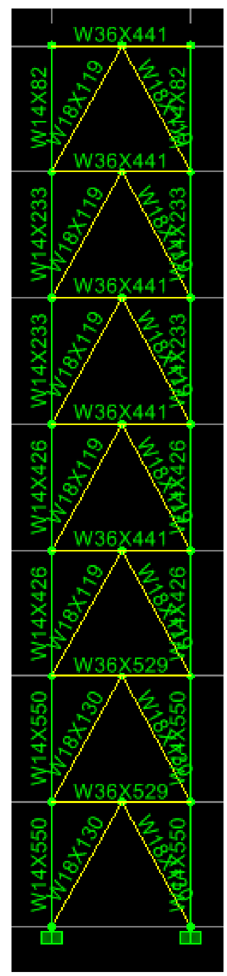
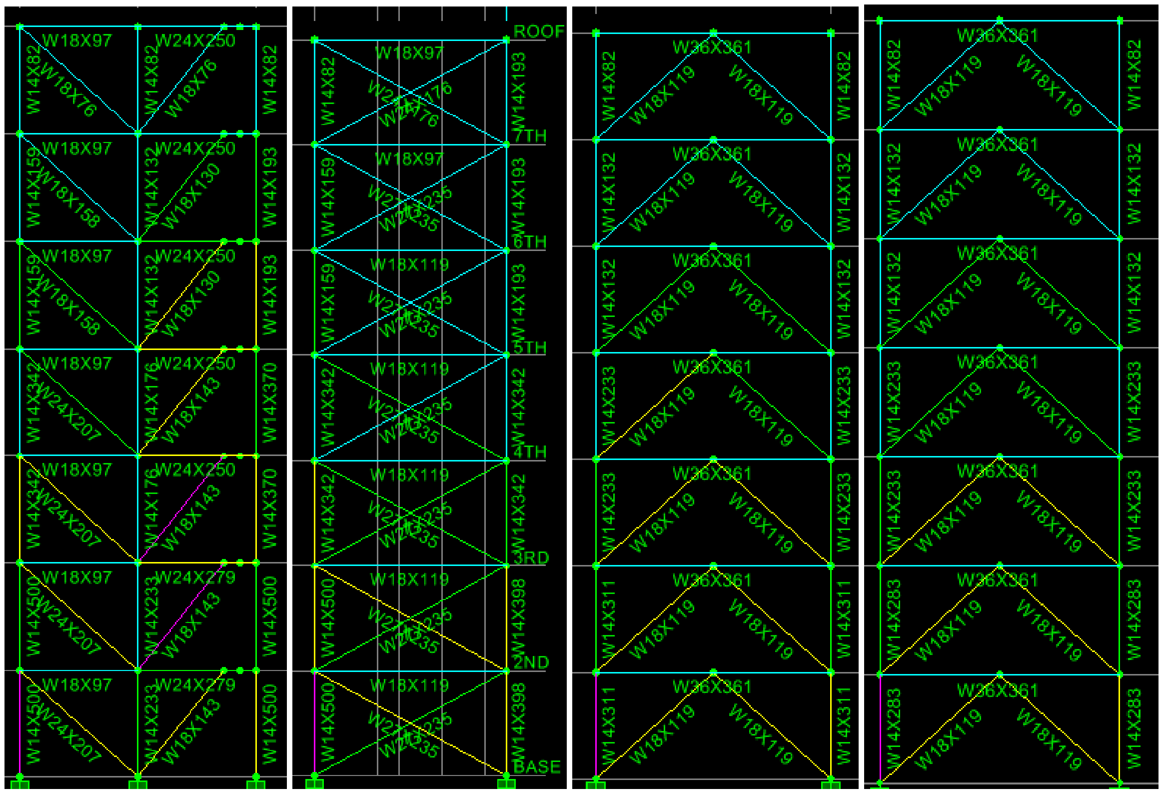


Figure 30 – NT-B and NT-D Elevation



VT-A *VT-B* *VT-D* *VT-E*

Figure 31 – Vertical Truss Elevations Under Controlling Loads

3.6.7.4 Continuing Design

The level of detail in this design was considered sufficient to move onto the architectural and mechanical breadths. Due to time constraints and the complexity of designing lateral systems to resist seismic shear, engineering of the lateral force resisting systems could not be carried further. The author recognizes the following items need to be engineered to develop a working lateral system that could be used in a building like Quantum III. Had more time been available, these items could have been investigated.

1. EBF beam design outside of the link
2. EBF and SCBF beam shear reinforcing design
3. EBF and SCBF connection details

Furthermore, the heavy beams used throughout inverted V-trusses in the current design are unacceptable. They are uneconomical and inefficient as are all inverted V-trusses in American Eagle Outfitters: Quantum III. For the next iteration, these frames should be modified into two story X-braced frames to achieve uninterrupted vertical load paths. Another option would be to add shear reinforcing to aid the web in resisting these large magnitude forces. As a result, the beam designs will decrease in size dramatically. Alternatives to NT-B and NT-D should also be

considered. Their lower rigidity in comparison to VT-A, B, D, and E not only makes them inefficient in terms of member sizes, but allows the diaphragm to rotate much more on the west side of the building relative to the east.

Eccentric braced frame beam links require shear reinforcing at the ends of the link and intermittently. A design of one instance of this was performed, but it was for a preliminary design not consistent with Design B.

3.6.7.5 Redundancy and Irregularities

Currently, the design does not contain any torsional irregularities. If the structure were to have this irregularity, the equivalent lateral force procedure would not be permitted to use in the design of Quantum III. The only irregularity the structure has is a re-entrant corner, requiring the increase of seismic forces by 25 percent for connection of diaphragms to vertical elements. The removal of one brace or connection within these frames does not reduce the strength of any story by more than 33 percent either. Therefore, the redundancy factor, ρ, remains 1.0.

3.7 Impact of Redesign

The addition of two floors in American Eagle Outfitters: Quantum III will change a number of factors throughout the structural system. Foundations will increase with larger building mass. Piles capacity can be increased, and their original capacity is outlined in 3.1.1 Geotechnical and Foundation Concerns. Gravity columns at the lower levels will increase as well.

As a result of the two additional floors, more wind and seismic overturning is present. With a high volume building like QIII, the factor of safety against wind overturning is large. In this case, it exceeds 60! Conversely, high volume buildings have higher mass each floor, lowering the factor of safety against seismic overturning. For Quantum III, the factor is only 10 against seismic overturning. This is still great enough to have no concerns of building overturning.

Finally, the new heating and cooling loads found in the mechanical system breadth require larger equipment on the roof. The original structural design was considered conservative its approach: two 35,000 pound units were expected to be placed on the roof. The structural system was designed for two 40,000 pound units in RAM Structural System. Since the building masses were obtained from this model, the impact of the new rooftop units is negligible to the equivalent seismic lateral forces and lateral and gravity design.

3.8 Structural Conclusion

The design was a success through providing the author with numerous design challenges never encountered in classroom work. Goals included learning the subtleties of seismic controlled lateral design. Considering the amount of detail this analysis went into, this was accomplished. Only a portion of design criteria were touched on because of the numerous detailing requirements in seismic regions. More so, this laid the foundation for the continuing education in lateral design that will be experienced in the workforce.